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Abstract

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ORIGINAL RESEARCH

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The use of polymeric and metallic geogrid on a full-scale MSE wall/embankment on hard foundation: a comparison of field data with simulation

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Abstract

A full-scale reinforced earth embankment was designed and constructed by the Department of Highways of Thailand on a hard foundation in Phitsanulok Province, Thailand. Two types of reinforcement were used in the embankment. One side was reinforced with polymeric reinforcement consisting of polyester (PET), polypropylene (PP) and high-density polyethylene (HDPE) and referred to as a reinforced steep slope (RSS), with an angle of 70° from horizontal. On the other side, the embankment was reinforced with metallic reinforcement consisting of metallic strips (MS) and steel wire grids (SWG) combined with vertical segmental concrete facing and referred to as a mechanically stabilised earth wall (MSEW). The behaviour of the reinforced soil slope and the mechanically stabilised earth wall on a hard foundation were observed and compared with predictions from the PLAXIS 3D software. The lateral displacements and settlements were very small in the case of the MSEW with inextensible reinforcement. The corresponding lateral and vertical deformations in the RSS were much larger due to its extensible reinforcing materials. The stiffnesses of the reinforcing materials decrease in the following order: MS, SWG, PP, HDPE and PET. The results obtained from three-dimensional (3D) finite element method simulations (using PLAXIS 3D) were in good agreement with the field measurements in terms of vertical and lateral deformations and strains in the reinforcement.

Keywords: Metallic reinforcement, Polymer reinforcement, Hard foundation, Test embankment

Background

In past years, many researchers studied the behaviour of several reinforced earth structures (i.e., mechanically stabilised earth wall/embankment) on Bangkok soft soil. Most of them were constructed in the premises of AIT campus. The fully instrumented steel grid reinforced embankment was constructed in the campus of Asian Institute of Technology in March 1989. The backfill of this reinforced embankment were clayey sand, lateritic and weathered clay whereas the reinforcement used was steel grid. Shivashankar [32] observed the behaviour of a welded wire wall with poor quality, cohesive-friction backfills on soft Bangkok clay.

Later on, In May, 1993, another embankment was constructed in AIT campus with polymer grid reinforcement as a reinforcing material. The reinforcement used in this embankment was Tenax TT 201 geogrid SAMP, which is uniaxial oriented polymer grid reinforcement [4]. Furthermore, Long [24] studied about the behaviour of a geotextile reinforced embankment on soft ground. This study was focussed on the interaction parameter between soil and geotextile reinforcement, the localized mobilization of geotextile reinforcement force related with slip failure, the performance behaviour of geotextile reinforced embankment on soft ground and the closed-form solution for rotational stability analysis of reinforced embankment. Similarly, Voottipruex [40] studied the behaviour of full scale embankment built in AIT campus which was reinforced with hexagonal wire mesh up to 6 m with 10° inclined of gabion facing. The facing consisted of large rectangular wire baskets wired all together and was filled with rock and height of each basket was 1 m. Two types of reinforcements, galvanized coated and PVC coated with unequal aperture size, were used in the two different sections along the length of the wall. Furthermore, Lai et al. [23] performed the full scale MSE embankment laid on fully instrumented Deep Mixing Method (DMM) improved ground. The behaviour of the full scale test embankment showed that deep mixing improvement method reduced the settlement of reinforced soil test embankment by 70%, which was an effective finding. Tanchaisawat [37] performed the study about the interaction between geogrid and tire chips-sand mixture, performances of full scale geogrid reinforced test embankment and numerical simulation of this full scale test embankment. The results revealed that the aperture sizes of geogrid affected most for the direct shear resistance of geogrids. He concluded that larger aperture size may lead to higher direct shear resistance.

The behaviour of a mechanically stabilised earth wall was studied by numerical simulation, laboratory testing and full-scale physical modelling. Many researchers have studied the behaviour of reinforced embankments, and most of them assumed plane strain conditions in doing so [5, 9, 10, 13–16, 18, 21, 31, 43]. The behaviour of reinforced embankments has also been investigated using three-dimensional (3D) finite element analysis [7, 8, 12, 34]. Smith and Su [34] reported that 3D finite element analysis can be used to model a reinforced soil embankment under service loading and at collapse. Briaud and Lim [12] utilised three-dimensional nonlinear finite element analysis to study the factors that influence the behaviour of tieback walls. Auvinet and Gonzalez [7] recommended that 3D analysis be considered under the following conditions:

- a. In the case of short slopes for which boundary conditions cannot be ignored.
- b. When the soil properties vary significantly along the longitudinal direction of the slope or embankment.
- c. When the slope is subjected to concentrated loading.
- d. When the potential failure is irregular.

Bergado and Teerawattanasuk [8] compared the effect of embankment geometry with two-dimensional (2D) and 3D simulations and concluded that 3D analysis must be conducted for short embankments to obtain good agreement with measured field data. Moreover, it was confirmed that geometric effects should be considered important factors that can affect the results of the numerical simulation. Huang et al. [19] has investigated different soil

constitutive models and their influences on the results. The paper confirmed that the modified Duncan–Chang model is a suitable constitutive model and that the parameters used in that model can be determined from conventional triaxial testing. However, the interface parameters used in their study were kept constant from the top to the base of the wall. Two- and three-dimensional numerical studies based on the finite element method (FEM) [3, 33] were used to analyse the deformation and influence of several parameters of reinforced soil walls of different types. Abdelouhab et al. [1] and Bourgeois et al. [11] investigated the influence of different types of synthetic strip reinforcement on the behaviour of a mechanically stabilised earth wall and identified the synthetic strip parameters that led to high horizontal displacements of the facing wall in the numerical simulation.

Moreover, Cisneros [17], Abiera [2], Mir [25], Kabiling [20], Modmoltin [26], Wong-sawanon [41], Srikongsri [35], Visudmedanukul [39], Asanprakit [6], Kongkikul [22], Supawiwat [36], Youwai [42], Rittirong [30], Prempramote [29], Tin [38] and Nualkiang [27] studied and analysed the behaviour of various reinforced earth embankment and its components during their research in AIT.

A full-scale test embankment was constructed in Phitsanulok Province, Thailand on hard ground using five different types of reinforcing materials (polymeric on one side and metallic on the other side). The reinforced steep slope (RSS) was reinforced with polymeric material with soil bags as the facing material, and the mechanically stabilised earth (MSE) wall was reinforced with metallic reinforcement with precast concrete panels as the facing material. This embankment was fully instrumented with piezometers, settlement plates, inclinometers, total pressure cells and strain gauges and subjected to careful field monitoring to obtain high-quality data. The aim of this study was to compare the behaviour of polymeric- and metallic-reinforced embankments on hard foundation with 3D numerical simulations conducted using PLAXIS 3D. Particular attention was given to the lateral displacements, vertical settlements, total vertical pressures and tensile forces in the reinforcement.

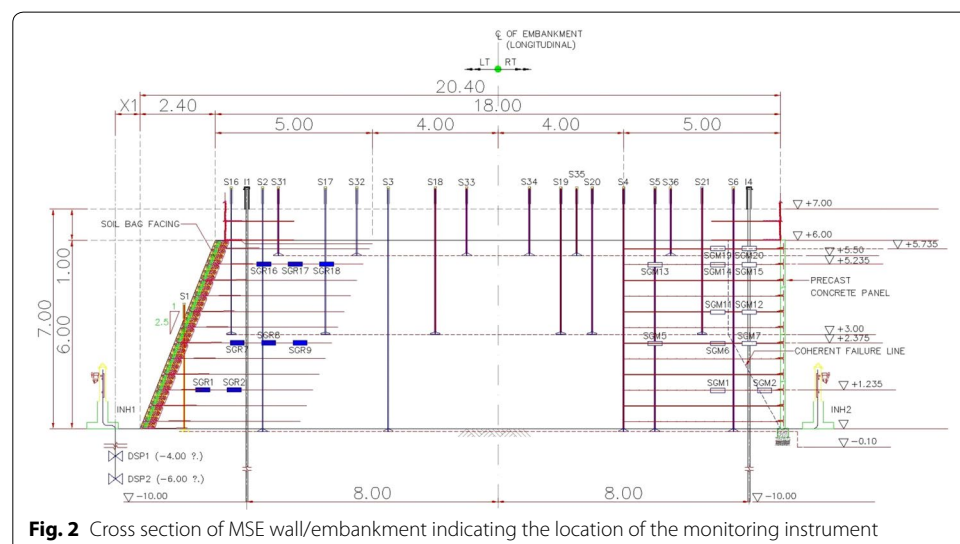
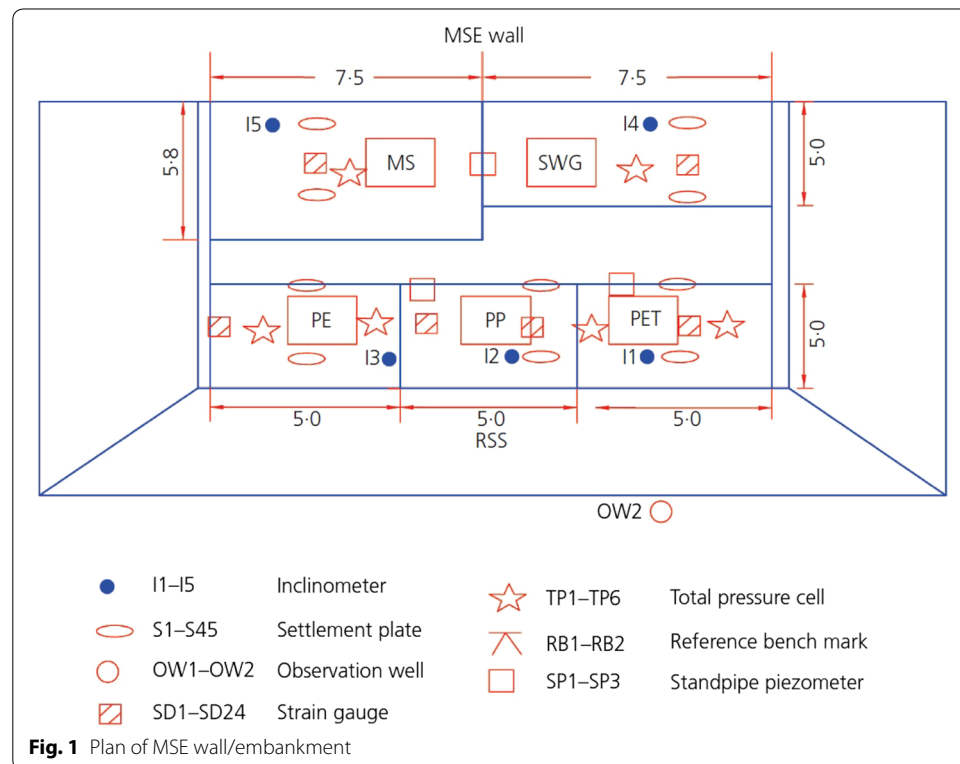
Description of the embankment

A 6-m-high reinforced earth embankment was constructed and designed by the Department of Highways (DOHs) of Thailand near Highway No. 11 in Phitsanulok Province in central Thailand. The test embankment was 18 m long and 15 m wide at the top. On one side, the reinforced steep slope (RSS) was constructed with a face sloping 70° from the horizontal and consisting of soil bags. On the other side, the mechanically stabilised earth wall (MSEW) was installed with vertical concrete panels as the facing. Three types of polymeric geogrids reinforcement were installed in the reinforced steep slope (RSS) facing, and two types of metallic reinforcement were installed in the mechanically stabilised earth wall (MSEW) facing. The three types of polymeric geogrid reinforcement were polyester (PET), high-density polyethylene (HDPE) and polypropylene (PP). The two types of metallic reinforcement were steel wire grids (SWG) and metallic strips (MS). The vertical spacing was 0.5 m and the length of the reinforcement was 5 m. The monitoring instruments installed to check the vertical and horizontal displacements, stresses, excess pore water pressures, depth to the groundwater table and strains in the reinforcing material included inclinometers, settlement plates, total pressure cells, standpipe piezometers, vibrating-wire strain gauges and fibre optic strain gauges. In

addition, two observation wells were installed to measure the fluctuation in the depth to the groundwater table. The plan and cross section of the MSE wall/embankment with the location monitoring instruments are shown in Figs. 1 and 2, respectively.

Construction of full scale MSE wall

Full scale MSE wall was constructed with full instrumentation program as outlined in “Description of the embankment” section. Preliminary design based on proposed wall geometry was carried out prior the execution of the construction work. Geogrid, steel



grid and steel strip as the reinforcement for the MSE wall was prepared in the desired length prior the installation work.

Preliminary design of MSE wall/embankment

Preliminary design of the MSE wall was carried out based on the external and internal stability analysis of the geometry of the wall. Limit equilibrium method of analysis was adopted for the preliminary design by assuming the interaction of the reinforcement with the backfill material based on the laboratory test data carried out in AIT. Silty sand mixed with lateritic soil (1:1 by volume) sourced from Phitsanulok is used as backfill material.

The wall height assumed for the preliminary design is 6 m with vertical facing. Vertical spacing of 5 (five) type of reinforcement adopted is 0.50 m and horizontal spacing for only steel strip is 0.50 m center to center of the 0.05 m wide strip of reinforcement. Precast concrete facing panel of 1.50 m height, 1.50 m width and 0.15 m thickness and with surface area of 2.25 sq m is used for the construction of the wall. Concrete strength of facing panel was 30 MPa.

The internal stability, tension in the reinforcement behind the failure surface was checked against the lateral internal earth pressures. The lateral earth pressure coefficient was assumed to be uniform throughout the height of the wall at the value corresponding to the at rest condition (K_0). The effects of compaction were ignored in this preliminary design. The total pullout resistance inclusive of the friction over the reinforcement behind the failure surface was checked to be greater than the driving forces due to the internal lateral earth pressure. Also, the maximum tension force in each layer of reinforcement at the peak pullout resistance was checked not to exceed its yield point.

The external stability was examined using the conventional Bishop's method of slope stability analysis. A conservative estimate of minimum factor of safety is assessed to be around 1.5. The soil strength was assumed to be fully mobilized along the potential failure surface. Factor of safety was defined as the maximum reinforcement force available divided by the force required for stability to give a factor of safety of unity.

Construction method

Site clearing and levelling works were carried out for the marking of the position of the proposed MSE wall/embankment. First course of pre-cast concrete facing panels were placed into position using lifting equipment. During the installation of the precast concrete facing panel, vertical and horizontal alignment of each panel was inspected using a spirit level. Adjustment of the verticality of the facing panel was carried out with the help of securing wooden wedges in between the facing panel. Clamp made of timber and steel rod complete with fastener was used to secure the positioning of each facing panel from movement.

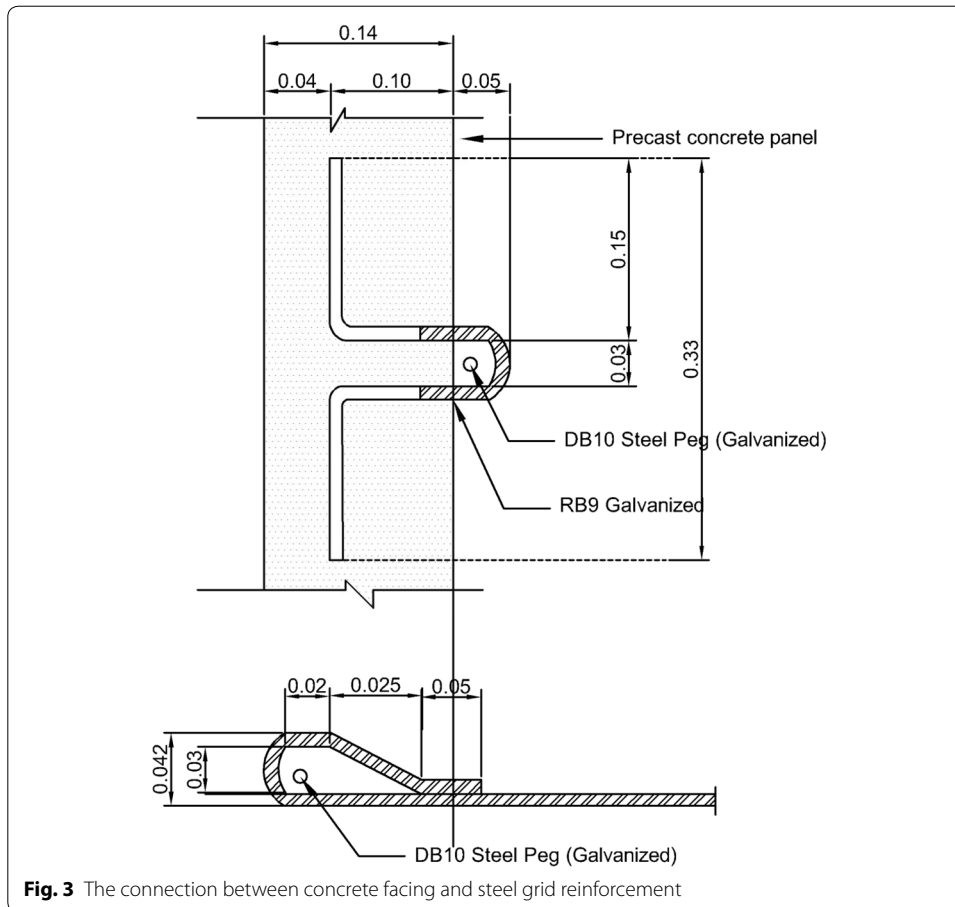
Geogrid reinforcement form delivered in roll form (approximately 100 m per roll) from the factory. It was cut by length of 7.0 m each and optical fiber with sensors was installed at designated locations of the selected geogrid reinforcement at the site.

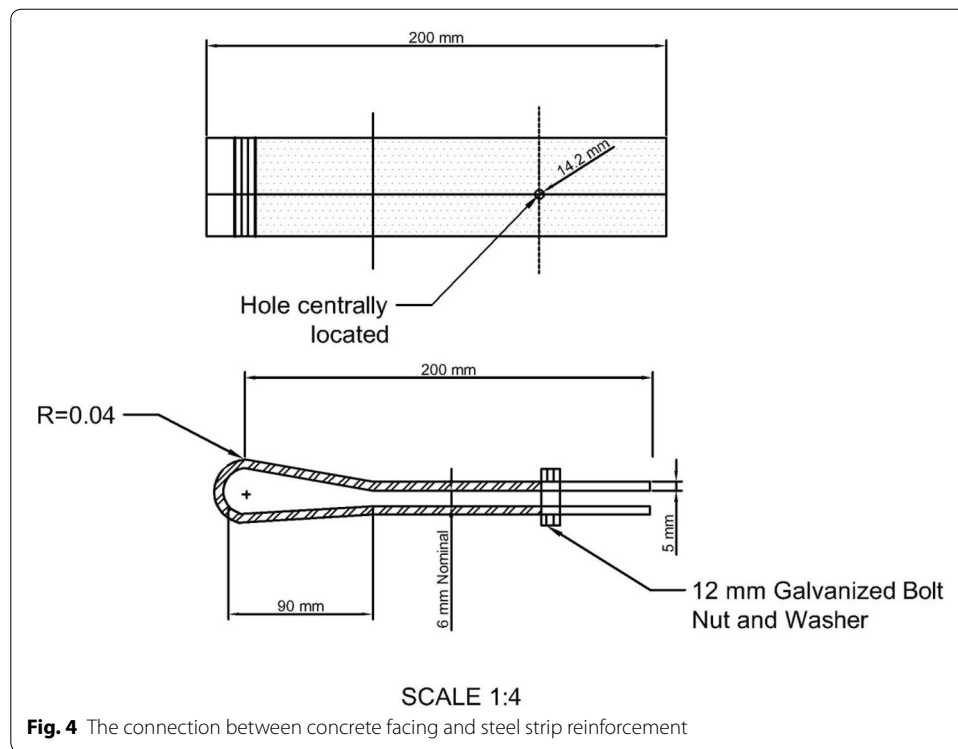
Steel grid and steel strip with 5.0 m length was installed and connected to concrete panel facing. Vibrating wire strain gauges were fixed to steel reinforcement at designated locations at the site.

The first course of backfill material (silty sand) was spread at the rear of the precast concrete facing panel. Compaction of backfill material to 95% proctor density of the

backfill material was carried out by a vibratory compactor. A small vibratory compactor was utilized to carry out the compaction work at narrow area with interference of the instruments installed. After the completion of the compaction work, (1) the first layer of steel grid reinforcement was installed into position and attached to the precast concrete facing panel by lap joint mechanism (Fig. 3). The lap joint mechanism was secured by inserting a 10 mm diameter deformed steel bar across the lap fold mesh. Steel grid was installed horizontally along the wall. The effective length of the reinforcements was 5.0 m after deducting for the connection. (2) The first layer of steel strip reinforcement was installed into position and attached to the precast concrete facing panel by using a 12 mm diameter galvanized bolt (Fig. 4). (3) The first layer of geogrid reinforcement was installed into position and turns up geogrid reinforcement at the face of the slope and returns the reinforcement a minimum of 1 m into the embankment below the next reinforcement layer (Fig. 5). This embankment was required soil bag with grass seed at the face to retain backfill materials (Fig. 6).

Subsequent course of backfill material measuring of 0.50 m thick were spread over the plan surface area covering the embankment. Similar compaction work was carried out before the next course of precast concrete facing panel was installed. The procedure as mentioned above was repeated until the full height of 6.0 m was achieved.





During the construction of the embankment, field density test at various selected places were carried out using sand cone replacement method to ensure compaction was carried out to minimum of 95% standard proctor density.

Model parameters

Foundation soils

The soil profile in Phitsanulok province generally consists of hard ground. One borehole (BH-1) was located in the middle of the embankment. Three additional boreholes were bored adjacent to the embankment near the RSS facing to obtain more data on the



Fig. 6 The soil bag facing to retain backfill material

soil profile for the 3D model of the embankment foundation. These additional boreholes were designated as BH-2, BH-3 and BH-4. The borehole locations are shown in Fig. 1. Figure 7 indicate the soil profiles corresponding to BH-1–4, respectively. The depth of the groundwater was 2 m below the ground surface. From Fig. 7, an abrupt change in soil profile was noticed which affects the settlement profile and deformation of the embankment too.

Backfill material

The backfill materials used in this embankment consisted of 50% lateritic soil mixed with 50% silty sand (by volume). The backfill material was classified as poorly graded sand (SP). It had an optimum moisture content of 7.8%, a maximum dry unit weight 19.62 kN/m^3 . A friction angle of 42° and cohesion of 80 kPa were measured in a direct shear test. Effective friction angles of 32.8° and 37° and effective cohesions of 0 and 20 kPa were measured in two different consolidated undrained (CU) triaxial tests. The properties of the backfill material are tabulated in Table 1.

Reinforcement

Two types of reinforcement, namely polymeric and metallic reinforcement, were used in the reinforced embankment. The polymeric reinforcement types used were high-density polyethylene (HDPE), polypropylene (PP) and polyester (PET). The metallic reinforcement consisted of metallic strips (MS) and steel wire grids (SWG). The properties of the reinforcement are tabulated in Table 2. The reinforcing materials used in the embankment are shown in Fig. 8. The MSEW facing with metallic reinforcement and the RSS facing with polymeric reinforcement are shown in Fig. 9.

Precast concrete panel

The vertical facing used in the MSEW portion of the embankment consisted of segmental precast concrete blocks 1.5 m by 1.5 m in size. The model parameters used in the numerical analysis for the precast concrete panels are summarised in Table 3. Interface element was placed in between soil element and concrete panel during simulation.

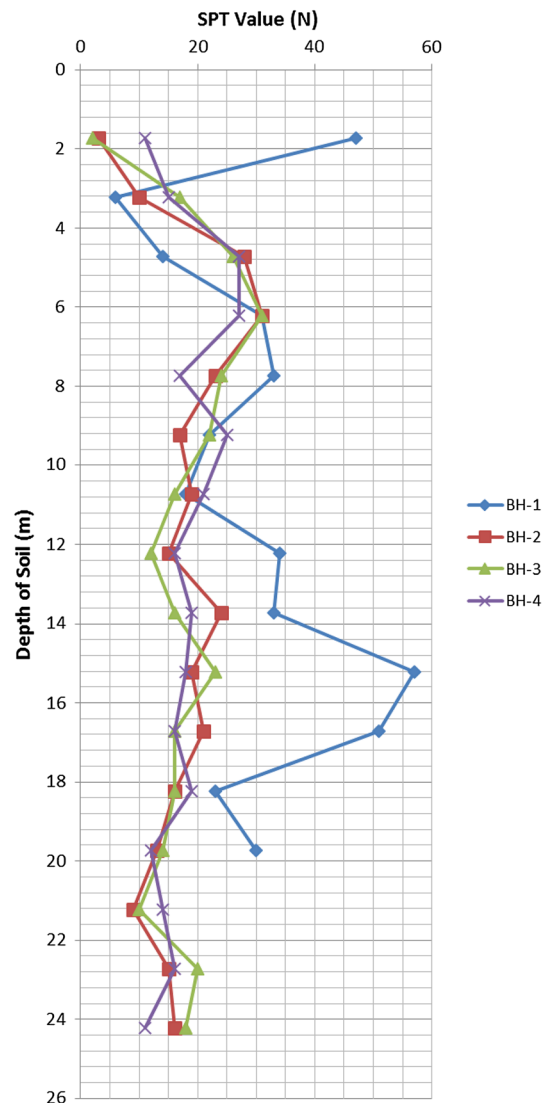


Fig. 7 Soil profile SPT value (considering BH 1–4)

Interface shear strength and large scale direct shear test

The interaction between the backfill soil and the reinforcing materials, namely, the metallic strips (MS), steel wire grids (SWG), polyester (PET), polypropylene (PP) and high-density polyethylene (HDPE), was assessed by conducting large-scale direct shear tests. The interface coefficients, R_{inter} , of the different types of reinforcing materials and the backfill soil shear strength are tabulated in Table 4. The large-scale direct shear tests were conducted under different normal stresses (i.e., 30, 60 and 90 kPa) and revealed that the maximum interface shear stresses between the steel wire grids and soil were highest, followed by the interface shear stresses between soil and soil, between metallic strips and soil, and between the three types of geogrids and soil. The details regarding apparatus and procedure of large scale direct shear test are described as below:

Table 1 Properties of backfill material

Atterberg limit test	LL = 20.8%, PL = 17.3%, PI = 3.5%
Sieve analysis test	Sample no. 1 Percent finer (#200 sieve) = 0.94%, Cu = 40, Cc = 0.34 Sample no. 2 Percent finer (#200 sieve) = 0.14%, Cu = 42.86, Cc = 0.55
Unified classification	Poorly graded sand (SP)
AASHTO classification	A-2-4(0)
Compaction test	Maximum dry density ($\gamma_{d, \max}$) = 19.62 kN/m ³ Optimum water content (OMC) = 7.8%
California bearing ratio (CBR) test	CBR = 50.5%
Direct shear test	Friction angle = 42° Cohesion = 80 kPa
Triaxial test (CU test)	
Test no. 1	Friction angle = 32.8° Cohesion = 0 kPa
Test no. 2	Friction angle = 37° Cohesion = 20 kPa

Table 2 Properties of reinforcing materials

Material name	Tensile strength (kN/m)	Thickness (mm)	Normal stiffness, EA (kN/m)	Model	Type
Metallic strip (MS)	277.6	4.00	88,000	Geogrid	–
Steel wire grid (SWG)	128.1	6.00	35,000	Geogrid	–
Polyester (PET)	83.6	1.50	925	Geogrid	Miragrid GX80/30
Polypropylene (PP)	91.9	1.45	1360	Geogrid	Secugrid 80/80 Q1
High-density polyethylene (HDPE)	85.8	1.91	1320	Geogrid	TT 090 SAMP

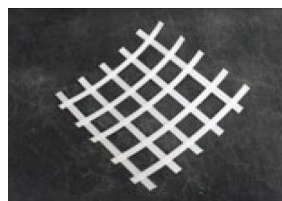
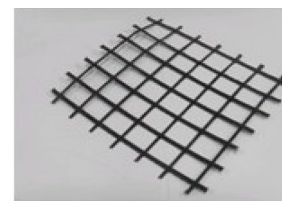
**PP****HDPE****PET****MS****SWG****Fig. 8** Reinforcing materials



Fig. 9 Photograph showing vertical face and sloping face showing metallic and polymeric reinforcements respectively

Table 3 Properties of concrete panel facing

Parameter	Name	Value	Unit
Type of behaviour	Material type	Elastic	
Normal stiffness	EA	42,000,000	kN/m
Flexural rigidity	EI	78,500	kN/m ² /m
Equivalent thickness	d	0.15	m
Weight	w	3.6	kN/m/m
Poisson's ratio	ν	0.15	–
Model		Plate	

Table 4 Interface coefficient (R_{inter}) and soil shear strength used in PLAXIS 3D

Soil to	Friction angle	Cohesion (kPa)	R_{inter}
Soil	40	23	1.00
Steel strip	36	23	0.87
Steel wire grid	40	28	1.00
Miragrid GX80/30 PET	33	21	0.79
Secugrid 80/80 Q1 PP	35	25	0.83
TT 090 SAMP HDPE	33	24	0.77

Apparatus

The large-scale direct shear apparatus were adapted from the pullout machine which is shown in Fig. 10. The lower half of the shear box was adapted from the empty pullout box with two 1.24-m-long steel channels placed along the both sides of the shear box. The cross-sectional dimensions of the steel channels were 9.5 mm \times 76.2 mm \times 152 mm. For the upper half of the shear box was composed of 9.5-mm-thick steel plate with an inside dimensions of 0.93 m \times 0.58 m \times 0.56 m, having roller bearings resting on the two bottom steel channels. Four steel bars with a diameter of 12.7 mm were welded in front of the upper box slightly above the predetermined shear surface. This allowed the upper shear box to be pulled by the same hydraulic jack used in the pullout machine. The top cover comprised of two 6.3-mm-thick steel plates with a pressurized air bag installed between them. The air bag was used for applying the normal pressures. Two steel angle beams were connected to the upper shear box that could run along its

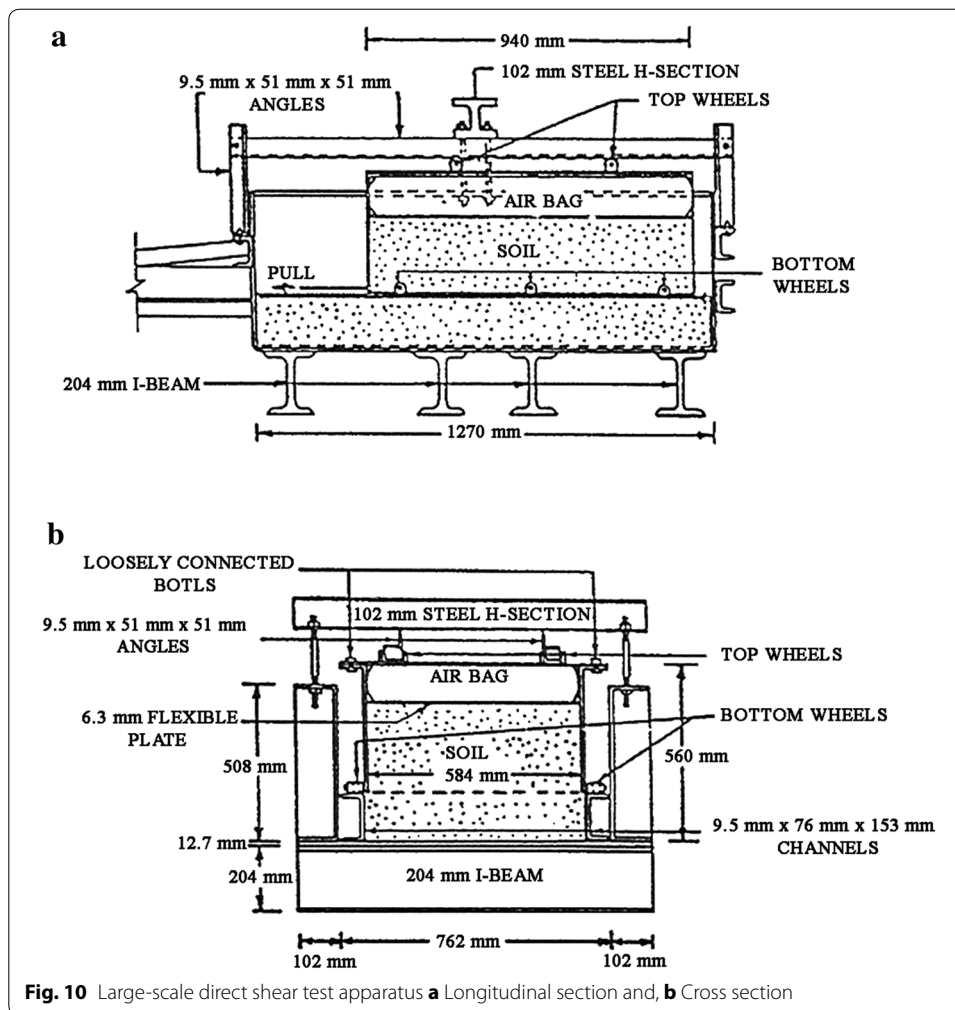


Fig. 10 Large-scale direct shear test apparatus **a** Longitudinal section and, **b** Cross section

horizontal direction by means of roller bearings. H-sectioned steel is also installed and connected to the top of the machine in order to form the reaction frame of the applied normal pressures.

Procedure

The predetermined amount of distilled water was added to the lateritic soil and silty sand 50:50 by volume. The water content was the same as the optimum moisture content obtained from the Modified Proctor compaction tests with the modified mold. The soil mixture put into the lower half of shear box and compacted until its surface was leveled to the height of box. This surface was the shear plane of the large-scale direct shear apparatus. The upper shear box with its roller bearings was next placed immediately above the lower sample, resting on the steel channels. The position of the upper shear box was fixed by using four C-shaped clamps. These clamps, with two steel angles placed across the top of the upper shear box, fastened the upper shear box to the sides of the lower shear box. This could prevent the rising of the upper shear box during compaction of another two succeeding layers of fill material, which was put into the box. The method

is used for controlling the density and the optimum moisture content. The flexible steel plate is laid on the top of compacted fill material, and then followed by the air bag and the top cover installation in order.

Numerical simulations

PLAXIS 3D Version [28] was utilised for the 3D FEM numerical simulations of the embankment. To minimise the effects of test embankment boundaries, the PLAXIS 3D discretisation was formulated and the boundary conditions were specified at distances of two times the length and width of the reinforced embankment in the x and y directions, respectively, as well as at a distance of four times the height of the reinforced embankment in the z direction. To carry out the finite element analysis of the embankment using PLAXIS 3D, a finite element mesh was created (Fig. 11), and the material properties of the embankment components were established (Table 5). The generation of an appropriate finite element mesh and the generation of properties and boundary conditions on an element level were automatically performed by the PLAXIS mesh generator based on the input of the geometry model. The 10 noded tetrahedron elements model in PLAXIS 3D consisted 101,325 elements and 148,547 nodes. In addition, the average size of the element was found to be 1.646 m. Soil properties as determined from tests on samples from four boreholes were used as the main inputs for the foundation soils in the FEM model. The water level was specified at 2 m below the ground surface. The polymeric and metallic reinforcements were characterised as geogrids, with their corresponding properties, whereas the precast concrete panels were characterised as plate elements. A surcharge equivalent to 1.2-m-thick fill was added at the top of the embankment 113 days after the start of construction. The simulation modelling was allowed to run for 186 days (125 days for construction and 61 days for consolidation analysis).

Staged construction

The staged construction for modelling the mechanically stabilised earth wall was divided into 16 stages with an equivalent period of 186 days. The embankment construction period with layers of compacted backfill lasted for 125 days and was followed by

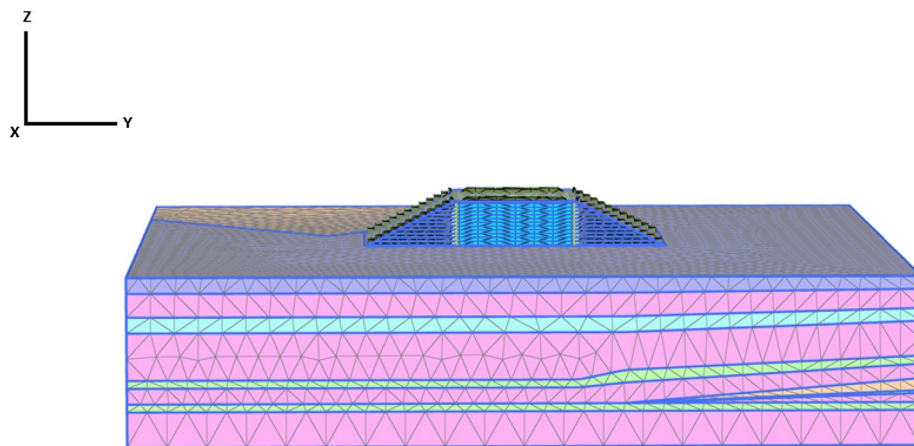


Fig. 11 3D discretisation model of MSE wall/embankment

Table 5 Material conditions and parameters used in PLAXIS 3D

Soil description	Model	Condition	γ_{sat} (kN/m ³)	γ_{unsat} (kN/m ³)	ν	E (kPa)	c' (kPa)	ϕ' (°)
Backfill	M-C	Drained	22.7	21	0.3	20,000	10	37
Loose clayey sand	M-C	Drained	19	17	0.3	18,000	1	33
Medium dense clayey sand	M-C	Drained	18	16	0.3	37,500	5	34
Stiff to very stiff clay	M-C	Undrained A	17	15	0.35	40,000	50	24
Very stiff clay	M-C	Undrained A	17	15	0.35	50,000	80	26
Hard clay	M-C	Undrained A	17.5	15.5	0.35	80,000	100	28

Undrained A uses the effective parameters for stiffness and strength in PLAXIS 3D

consolidation for 61 days. The staged construction and consolidation stages were modelled by placing the backfill material, along with the corresponding reinforcement, at incremental depths of 0.5 m per stage, until the embankment reached its full height, followed by 0.25 m of cover. The embankment was then loaded with a surcharge equivalent to a 1.2-m-thick fill 113 days after the start of its construction, and consolidation was allowed to proceed for 61 days. The individual stages and their durations are tabulated in Table 6. The loading time curve is plotted in Fig. 12.

Results

Lateral deformations

The lateral deformation of each type of polymeric reinforcement (i.e., PET, PP and HDPE) on the RSS side and each type of metallic reinforcement (i.e., MS and SWG) on the MSEW side obtained from field measurements using inclinometers is compared with the numerical simulation results at 186 days after the end of the construction. Inclinometers I1, I2, I3, I4, and I5 refer to the inclinometers installed in the PET, PP, PE, SWG and MS, respectively. Figures 13, 14, 15, 16, 17 show the measured and simulated lateral deformations of the PET, PP, PE, SWG and MS reinforcement, respectively. Large lateral

Table 6 Construction sequence of full-scale test reinforced embankment

Phase	Duration (days)
Initial	0
Phase 1	10
Phase 2	12
Phase 3	12
Phase 4	12
Phase 5	12
Phase 6	11
Phase 7	7
Phase 8	7
Phase 9	7
Phase 10	7
Phase 11	7
Phase 12	8
Cover 25 cm	1
Surcharge (1.2 m fill)	12
Consolidation	61

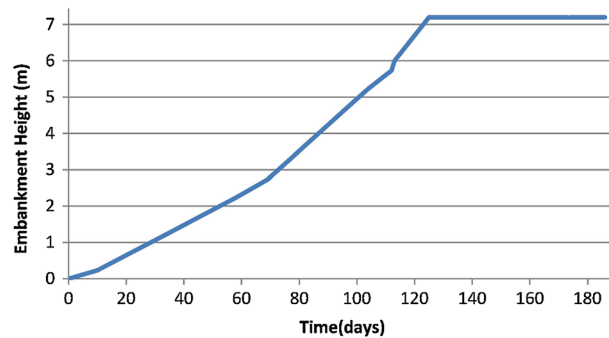


Fig. 12 Construction details of MSE wall/embankment

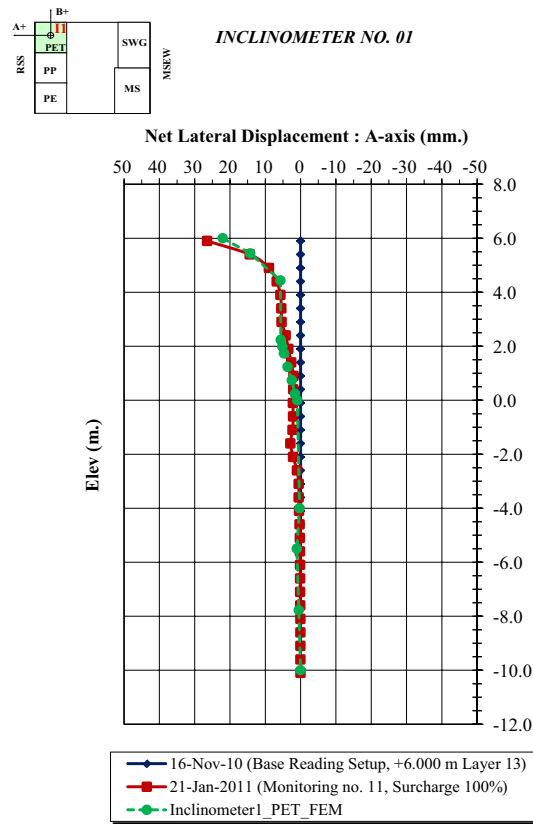
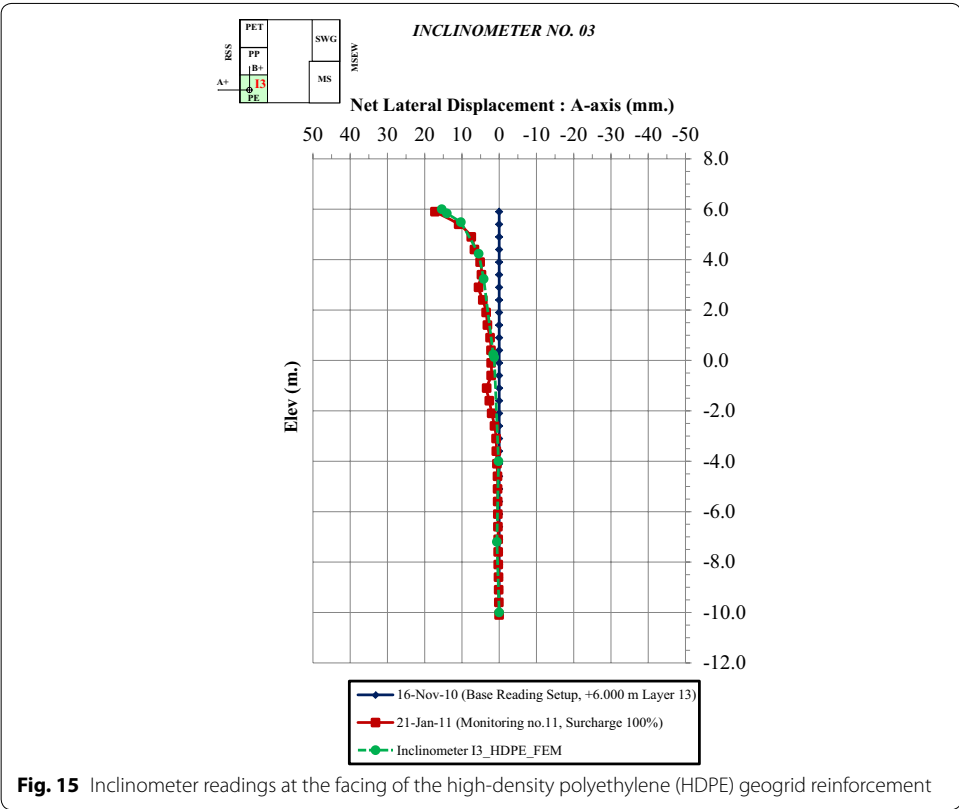
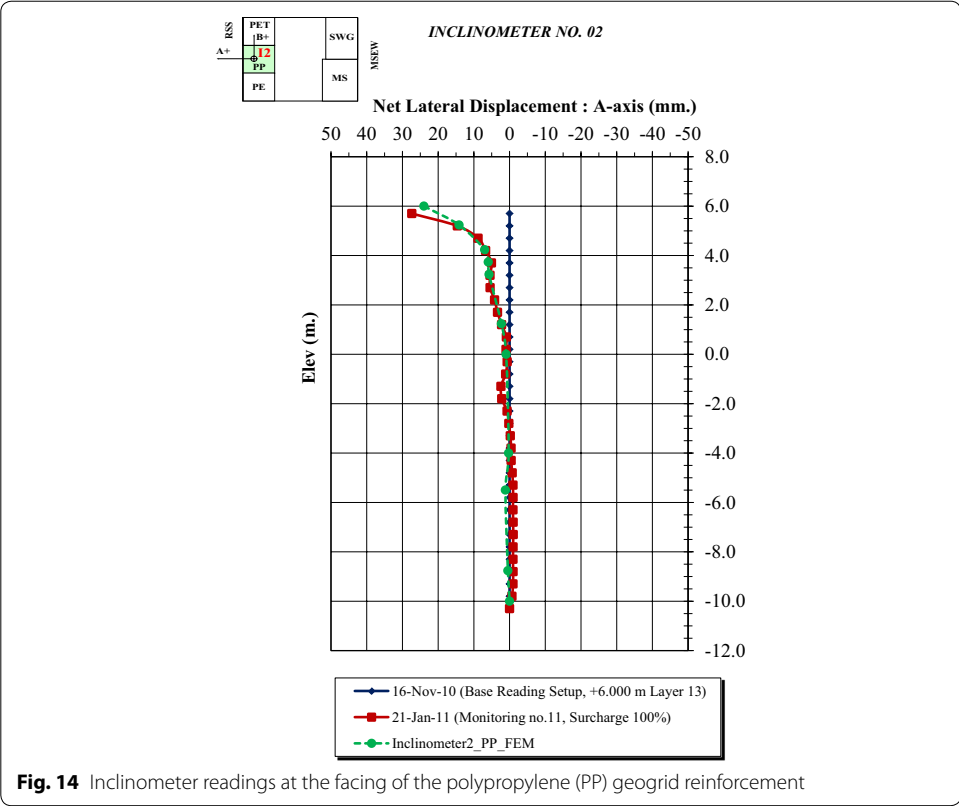
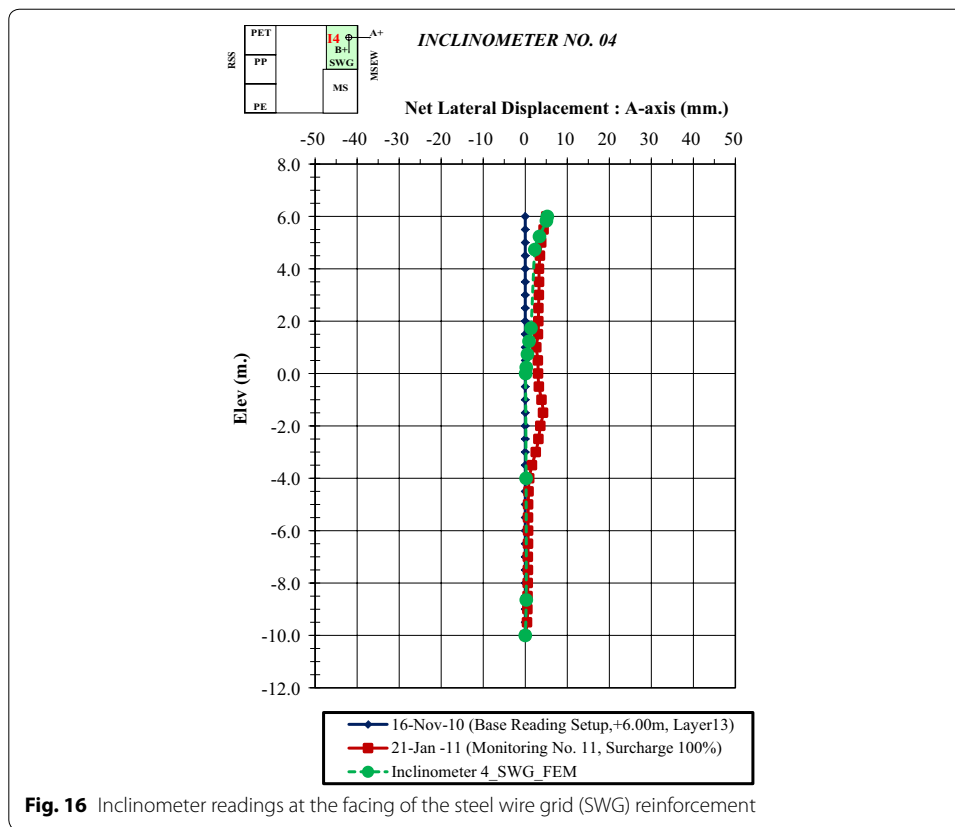


Fig. 13 Inclinator readings at the facing of the polyester (PET) geogrid reinforcement

displacements at the top of the PP, PET, and HDPE geogrid reinforcement at 186 days were noted after the 1.2-m-thick surcharge was added, which can be observed in the field too, as a tilt of the inclinometer on the top of the RSS facing (see Fig. 18). The largest displacement occurred in the middle of the RSS facing with the PP geogrid reinforcement. In the MSEW facing, the lateral displacement for SWG was slightly larger than for MS because its stiffness is lower than MS. In general, the simulation results agreed well with the field observations and polymeric reinforcements were successfully simulated than metallic reinforcements in terms of lateral deformation.



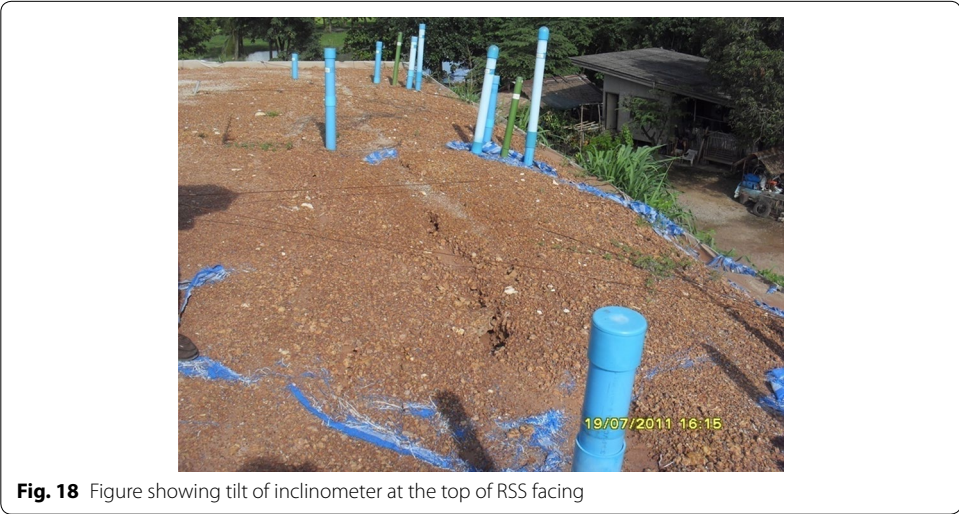
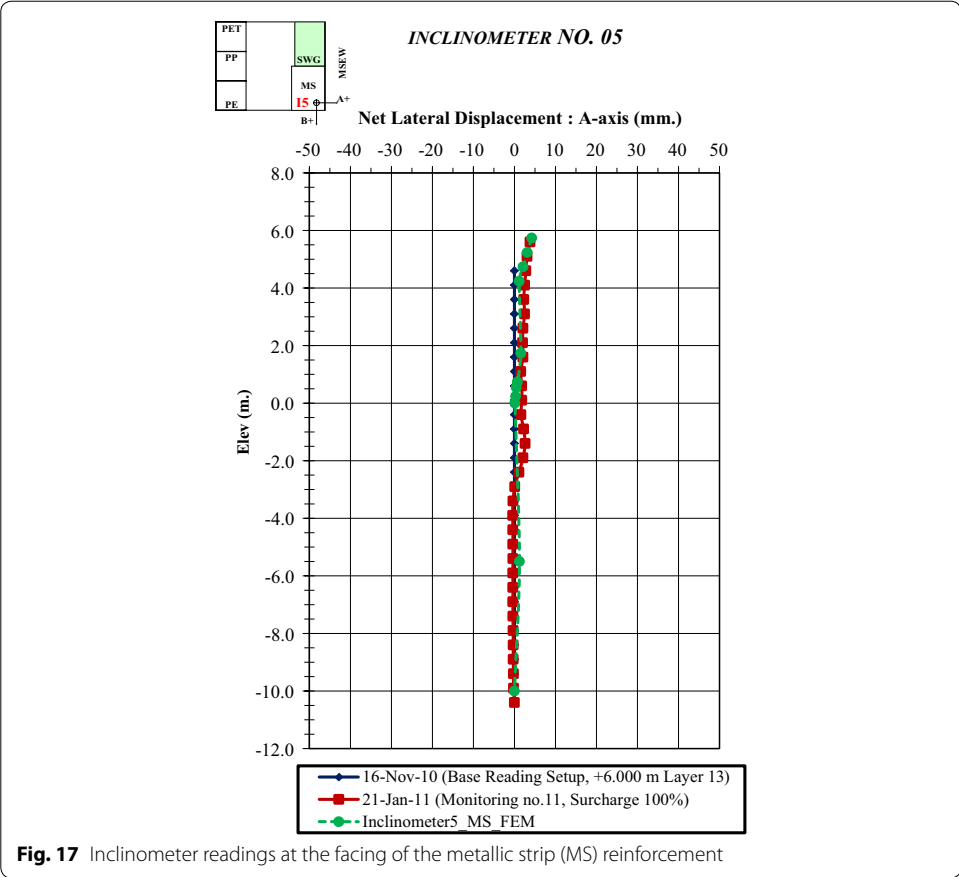


Vertical deformations

Surface and subsurface settlement plates were installed in the embankment at different locations to measure vertical settlements. The maximum settlement at the base of the embankment (Level 0.00 m) ranged from 60 to 80 mm 186 days after construction. The compression of the foundation was found to increase slightly towards the facing, as shown in Figs. 19a, 20a and 21a for the PET-SWG, PP-MS and PE-MS sections, respectively. Similarly, the compression of the embankment (Level 0.00 m to Level 5.50 m) varied between 20 and 40 mm, as shown in Figs. 19b, 20b and 21b for the PET-SWG, PP-MS and PE-MS sections, respectively. Due to the hard ground foundation, the magnitudes of the vertical settlements were relatively low. The settlement profile of the PET-SWG, PP-MS and PE-MS sections at different levels of the embankment (0.00 m at the base of the embankment and 5.50 m at the top of embankment) are plotted together with the simulated data in Figs. 19a,b, 20, 21a,b, respectively. Overall, the simulation results are consistent with the vertical deformations measured in the field.

Strains in reinforcement

The measured and simulated strains in the metallic and polymeric reinforcement were in good agreement. The strains were measured using vibrating-wire strain gauges in the metallic reinforcement (SWG and MS) and using fibre optic strain gauges in the polymer geogrids (PET, PP and HDPE). For the metallic reinforcement, the strains were lower than the geogrid strains because the metallic reinforcement was much stiffer



than the polymer geogrids. The measured and simulated strains in the metallic strips (MS) and steel wire grids (SWG) are plotted in Figs. 22 and 23, respectively, after 125 and 186 days. The corresponding strains in the polymer geogrids (PET, HDPE and PP) are plotted in Figs. 24, 25, 26, respectively. In general, good agreement was observed

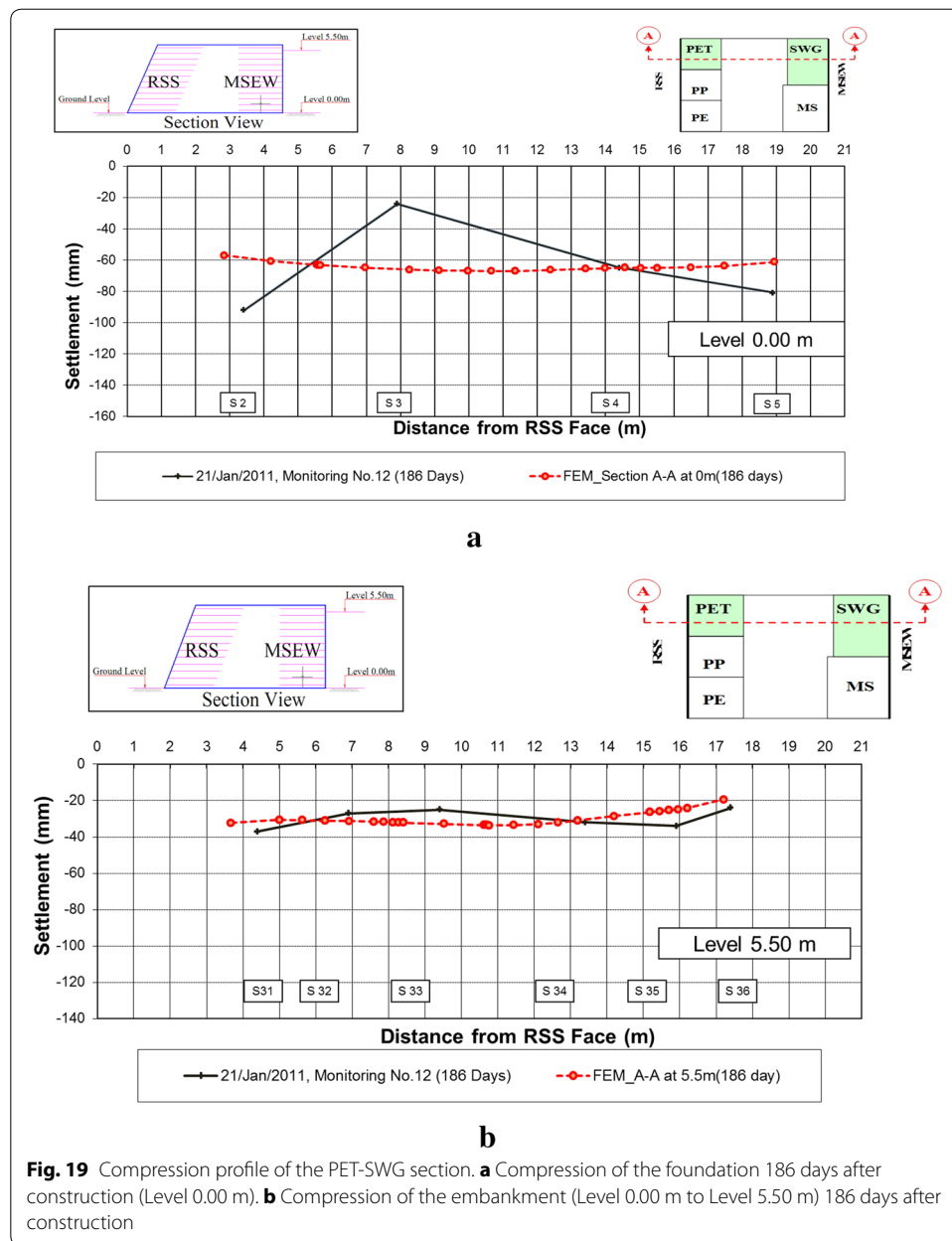
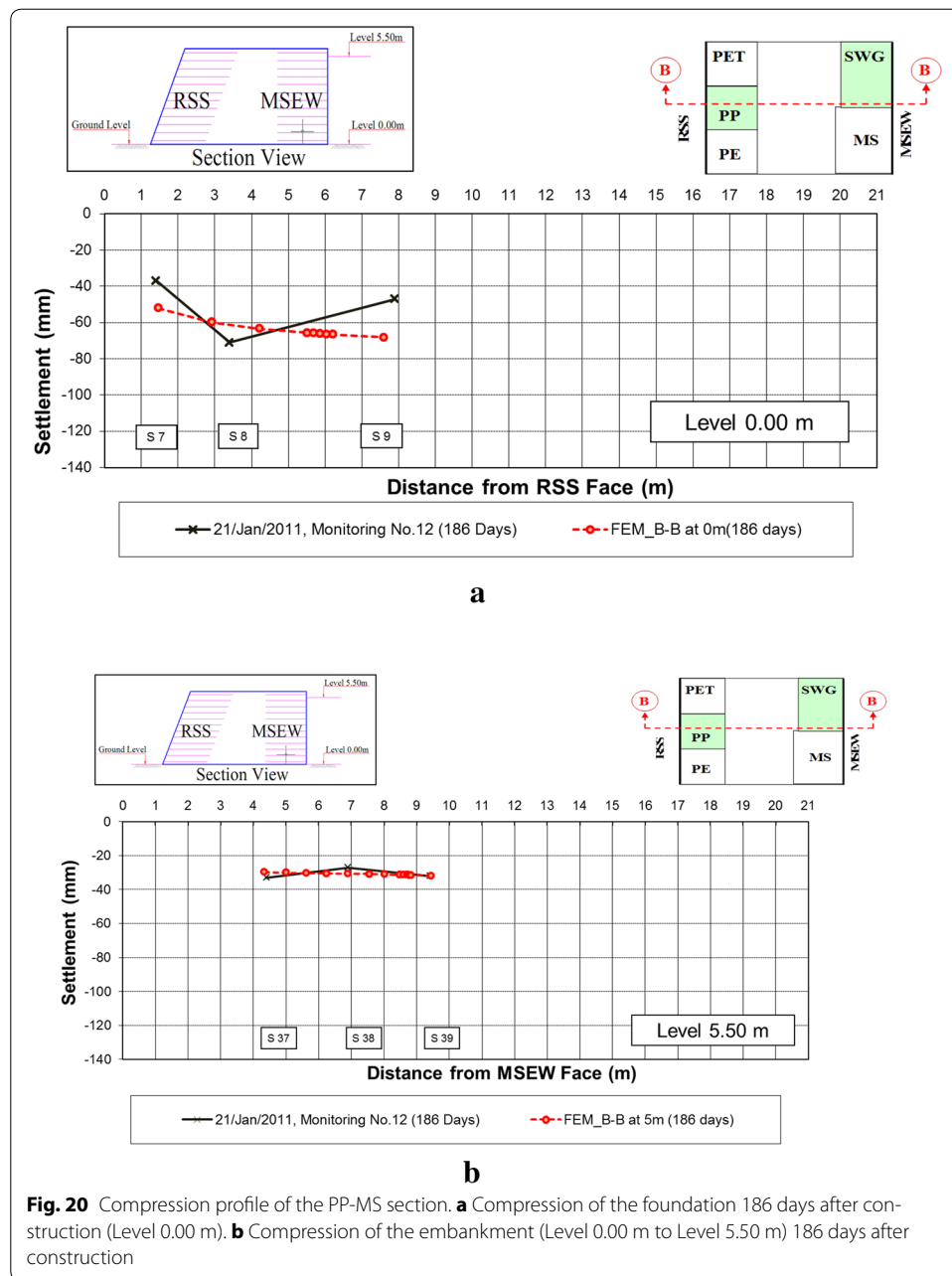


Fig. 19 Compression profile of the PET-SWG section. **a** Compression of the foundation 186 days after construction (Level 0.00 m). **b** Compression of the embankment (Level 0.00 m to Level 5.50 m) 186 days after construction

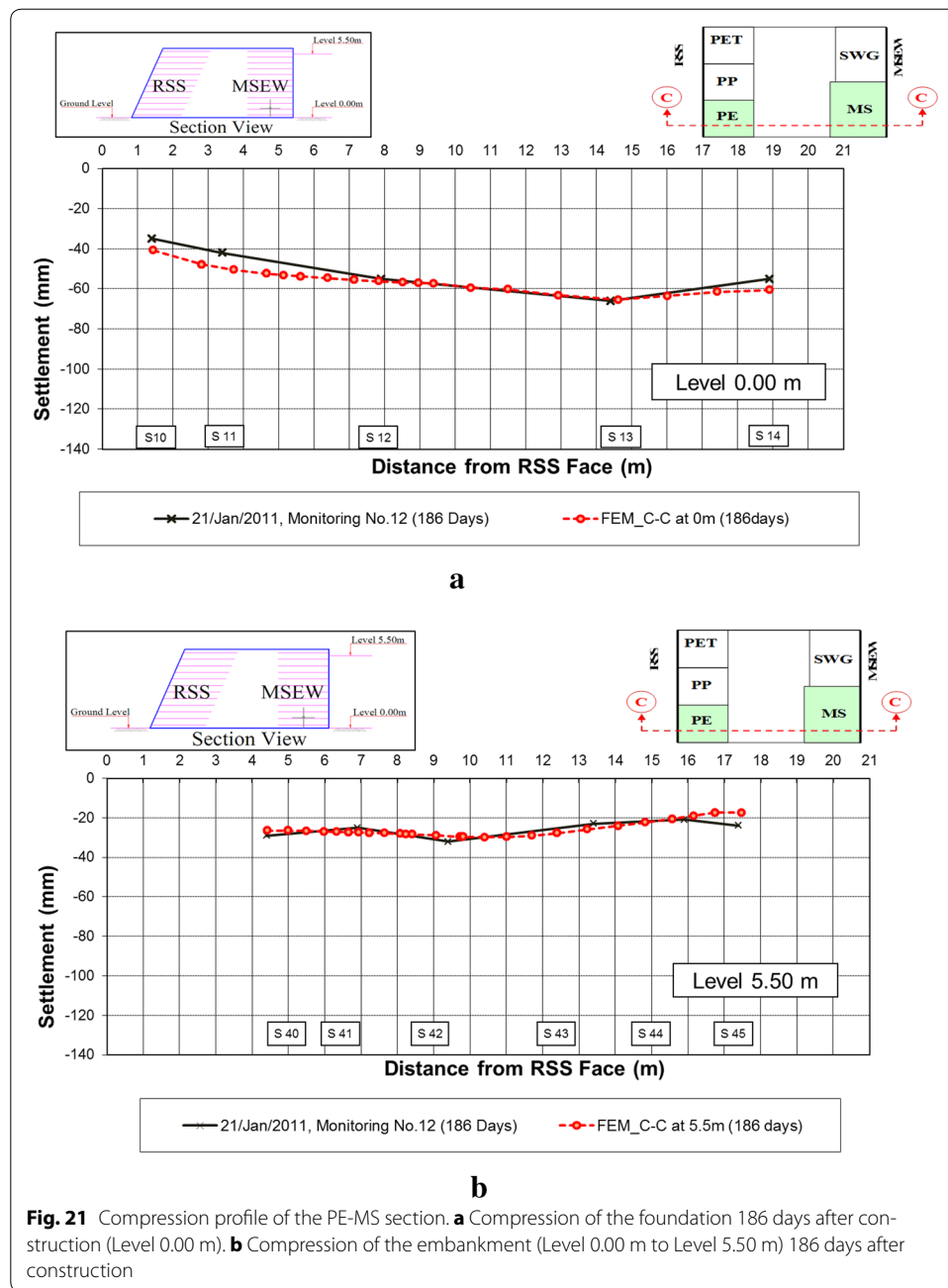
between the measured and simulated strains. The line of maximum strain corresponded to bilinear rather than linear behaviour.

Total pressure cells

Total pressure cells were installed at various locations at the base of the embankment at level 0.00 m (see Fig. 27). The total pressure cell TP2 indicated that the highest total pressure measured was 275 kPa after 186 days, while the maximum total pressure recorded by TP5 was 175 kPa. These values are abnormally higher than the weight of the embankment. The pressures measured by TP1, TP3, TP4 and TP6 ranged from 130 to 150 kPa, which are consistent with the weight of the embankment. The variation in

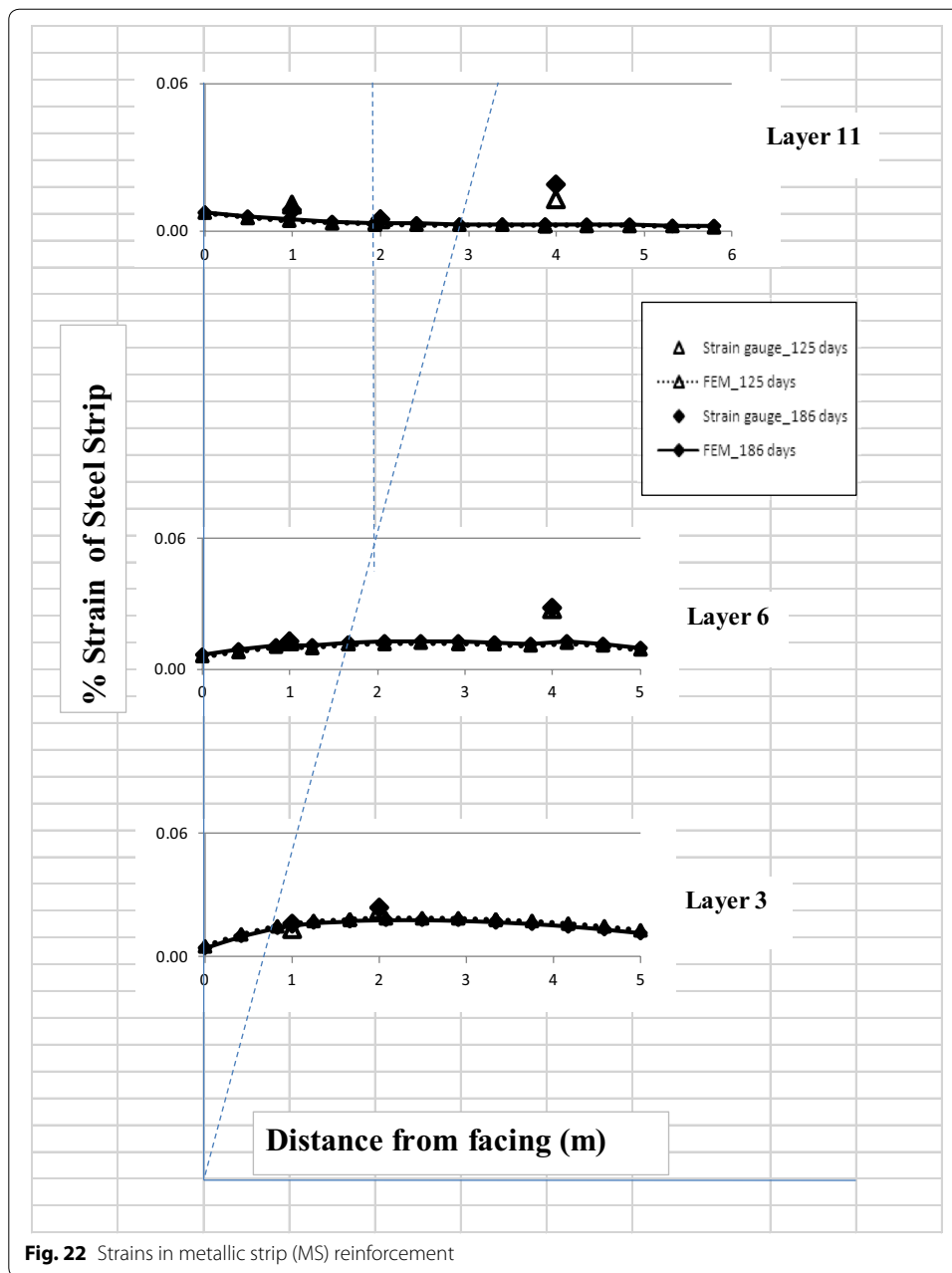


the total pressures measured by the six pressure cells at ground level are compared with the results from the 3D simulations in Fig. 27. There was an unexpectedly high value recorded for TP2, which might have been due to some problem with the instrumentation. This issue can be confirmed by plotting the polynomial distribution of total pressures from each of the pressure cells with respect to embankment height, as shown in Fig. 28. The total pressures after surcharge and after consolidation were almost constant and good agreement between the measured and simulated values were obtained.



Discussions of the results

The field data obtained from full-scale reinforced embankment are compared with the results obtained from FEM 3D simulation of embankment. PLAXIS 3D (ver. 2011) was used as a tool for FEM Simulation. Comparisons were mainly made on lateral and vertical displacement, strains in the reinforcements and total pressures. The horizontal field deformation indicated by inclinometers for all polymeric and metallic reinforcements agreed well with simulation data and larger deformation were noticed for successfully simulated polymeric reinforcements in comparison with metallic reinforcements due to higher stiffness of the metallic reinforcements. In addition, larger deformation was



observed on the top of the embankment after the 1.2-m-thick surcharge has been added, which was confirmed by tilting of embankment on topmost part in field too as shown in Fig. 18. Similarly, the measured vertical compression data for each section are compared with simulated data and it was found that vertical settlement was found to be varied from 60 to 80 mm. Furthermore, the vertical compression of the embankment (Level 0.00 m to Level 5.50 m) was found to be varied between 20 and 40 mm (see Figs. 19, 20, 21). The value of settlement is found to be low as the embankment was constructed on hard ground.

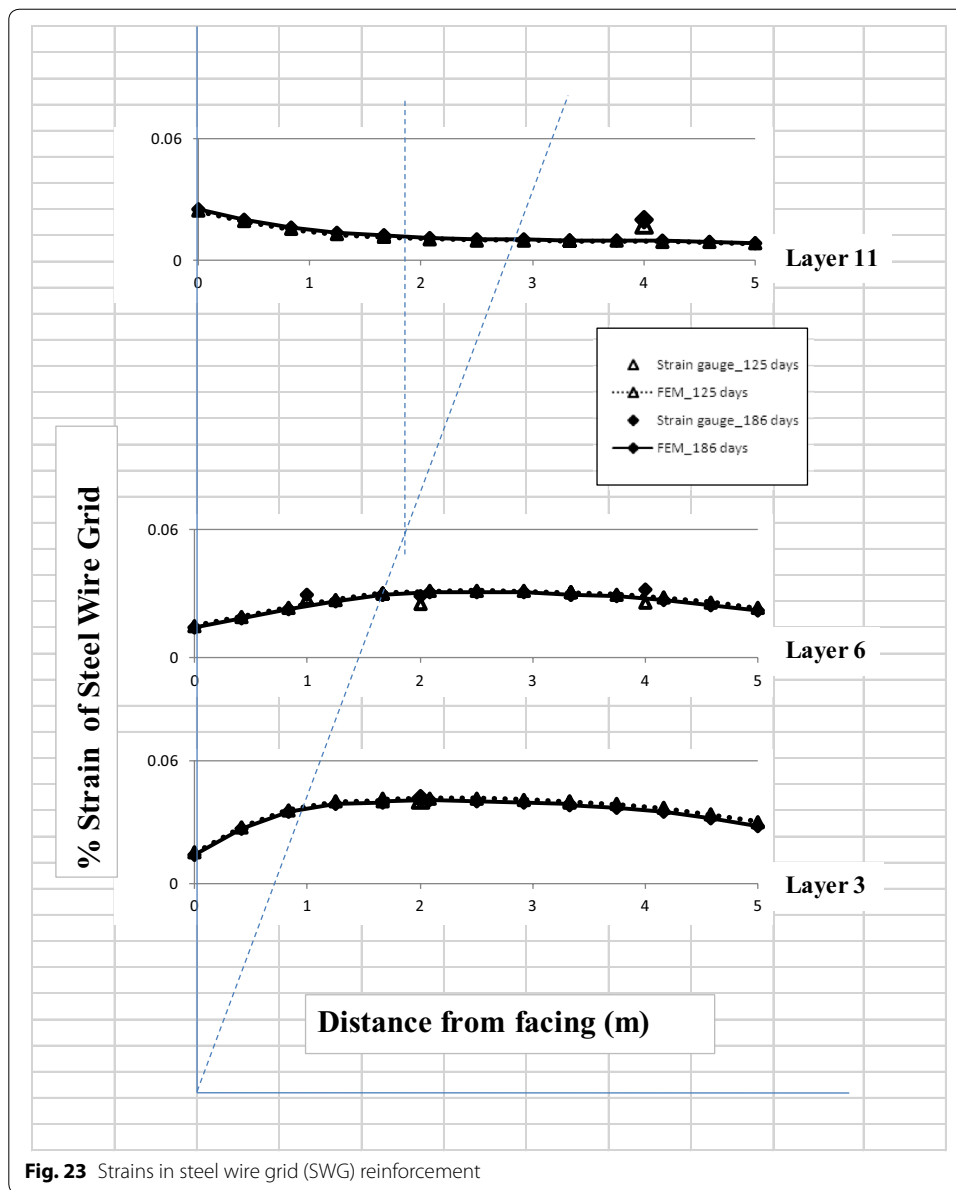


Fig. 23 Strains in steel wire grid (SWG) reinforcement

Vibrating wire strain gauges for metallic reinforcements and fibre optic strain gauges for polymeric reinforcements are installed on the embankment and results from field measurement were compared with the simulation results. Theoretically, the strains should be zero at the end of embankment (at a distance of 5 m from the facing) but due to the difficulties and inaccuracies associated with sensitive strain gauges, there is certain value of strain at the end of reinforcements (see Figs. 22, 23, 24, 25, 26). In general, good agreement was observed between simulated and measured strains and the line of maximum strain for both the metallic and polymeric reinforcements exhibited bilinear behaviour as expected.

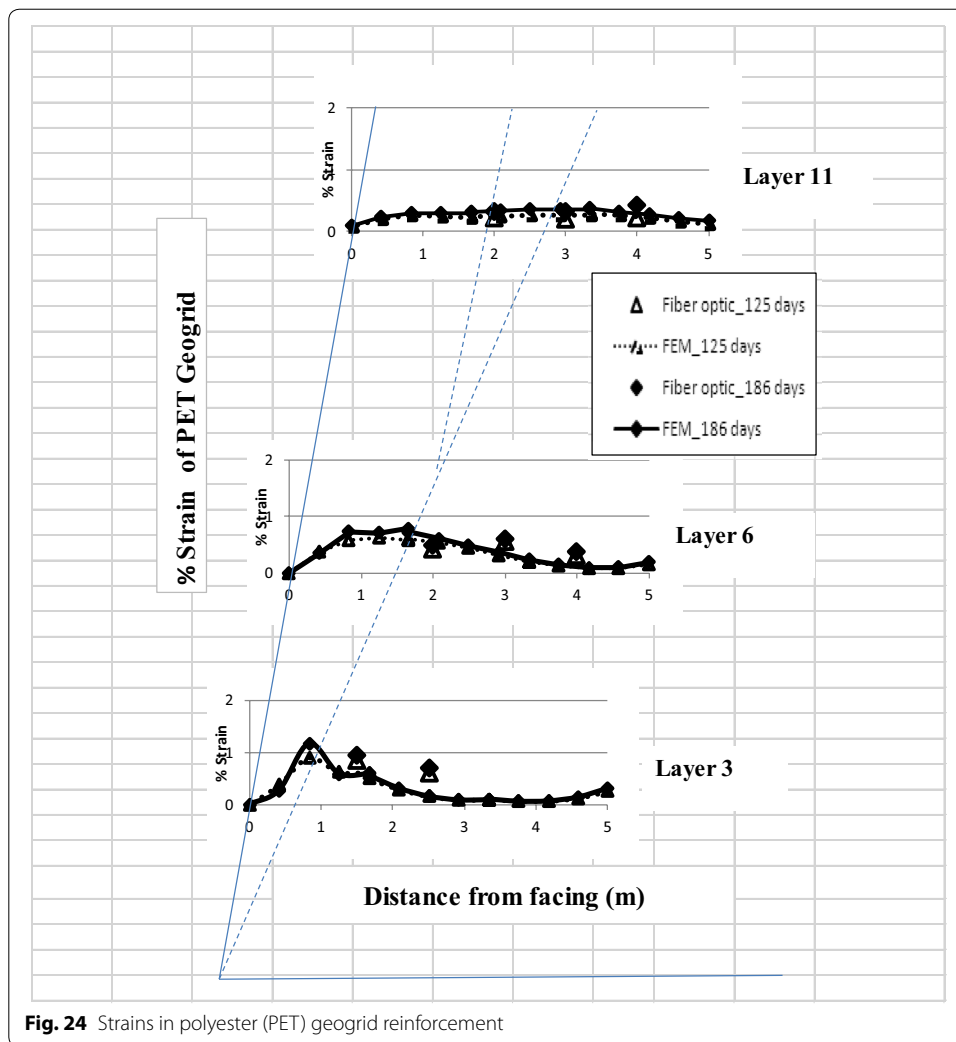


Fig. 24 Strains in polyester (PET) geogrid reinforcement

Six total pressure cells are installed at the base of embankment in order to measure total pressure at 125 and 186 days after construction and compared with the simulated data. It was found that the pressures varied from 130 to 150 kPa (see Fig. 27). Unexpected high value (i.e., 275 kPa) was noted by TP2 and might have been due to some instrumentation problem which was confirmed by plotting polynomial distribution of total pressures with height (see Fig. 28). Overall, the simulation results are consistent with measured field data.

Conclusions

A full-scale reinforced embankment was designed and constructed on a hard foundation in Phitsanulok, Thailand, with polymeric reinforcement on one side and metallic reinforcement on the other side. The metallic reinforcement in the mechanically stabilised earth wall (MSEW) facing consisted of metallic strips (MS) and steel wire grids (SWG). The polymer reinforcement in the reinforced steep slope (RSS) facing consisted

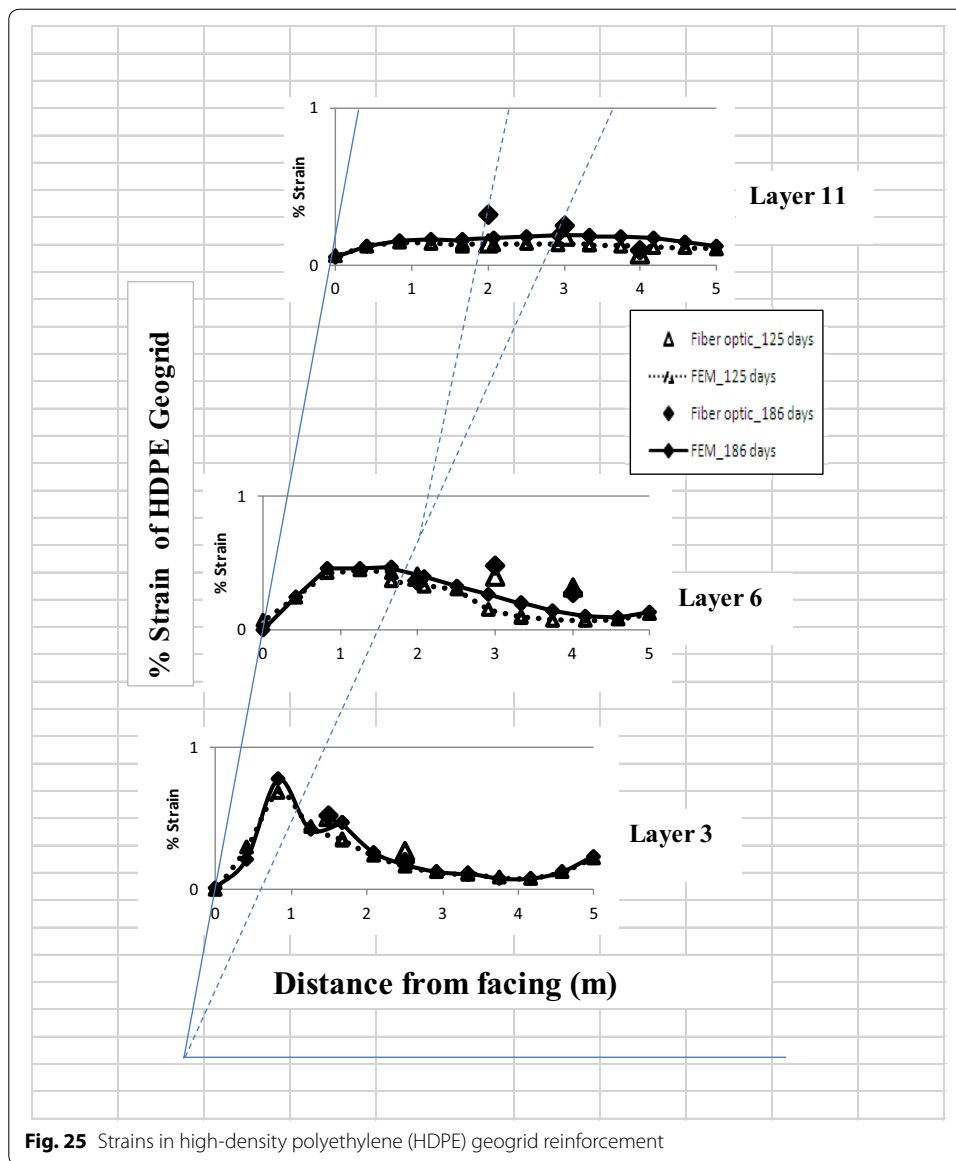
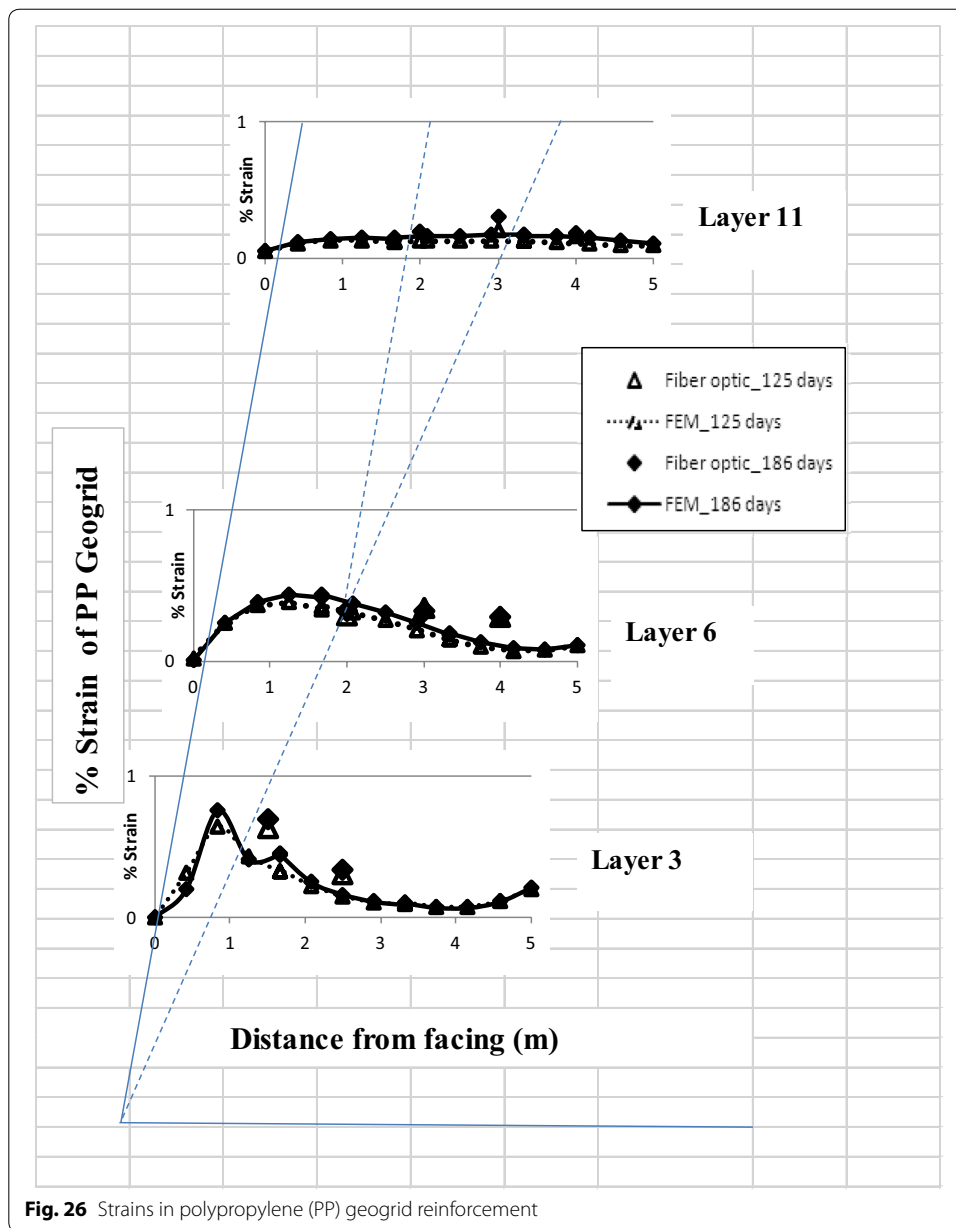
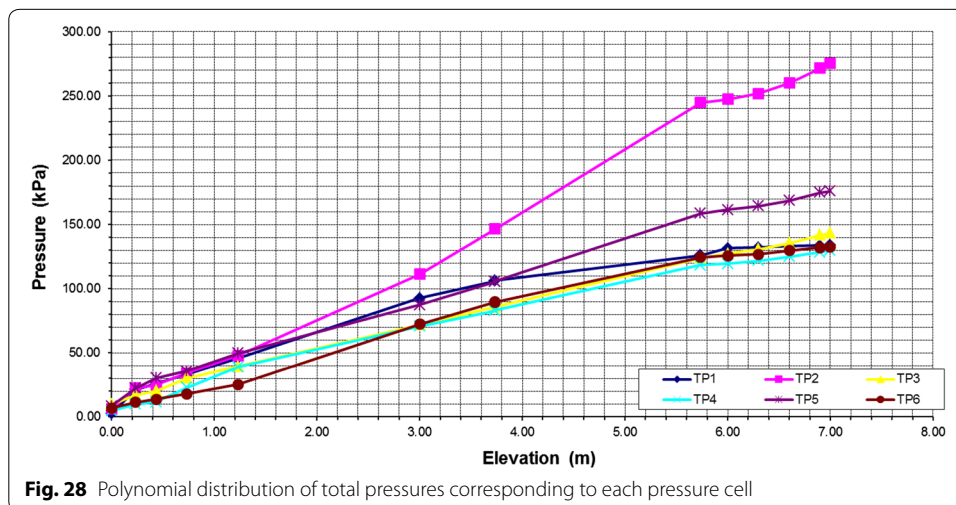
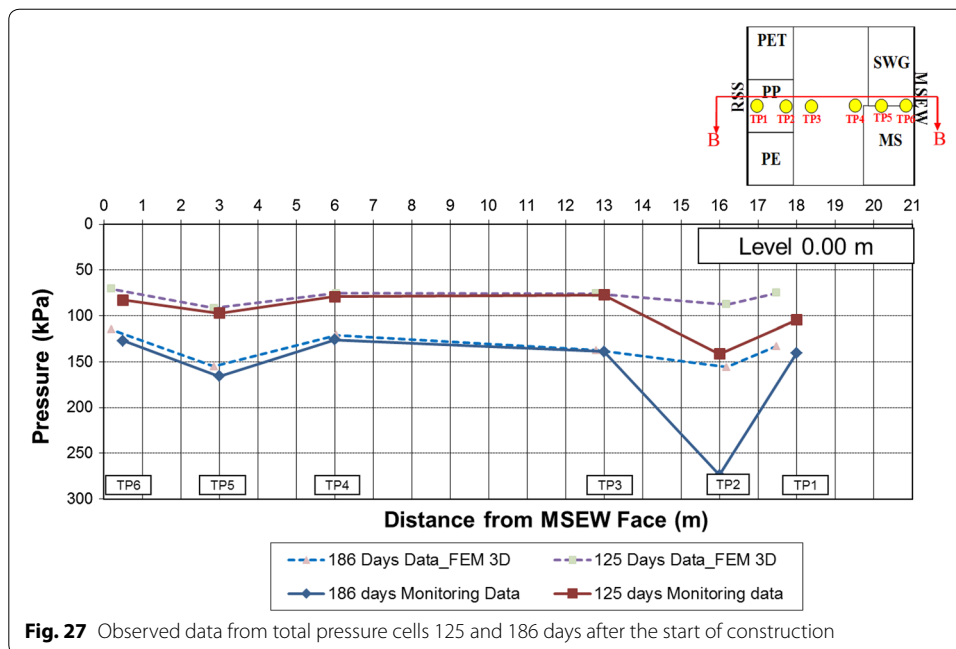


Fig. 25 Strains in high-density polyethylene (HDPE) geogrid reinforcement

of polypropylene (PP), high-density polyethylene (HDPE) and polyester (PET) geogrids. The behaviour of both the metallic and polymeric reinforcement was monitored and observed. The lateral and vertical embankment deformations in the MSEW facing were very small, according to the field monitoring results. The deformations of the RSS facing were much greater than those in the MSEW facing because the polymeric reinforcement was not as stiff as the metallic reinforcement. The reinforcing materials can be listed in the following descending order in terms of stiffness: metallic strips (MS), steel wire grids (SWG), polypropylene (PP), high-density polyethylene (HDPE) and polyester (PET). The measured lateral and vertical deformations for both facings, with the different types of reinforcement, agreed well with those predicted from the numerical simulation using PLAXIS 3D. The data from the total pressure cells showed that the total pressures after



surcharge (125 days) and after consolidation (186 days) were almost the same and that the measured and simulated values were in good agreement, except for the total pressure measured by cell TP2, which was found to have an instrument error. Furthermore, the strains in the metallic reinforcement and polymeric reinforcement agreed well with the strains predicted from the simulation. The line of maximum strain for both the metallic and polymeric reinforcement exhibited bilinear behaviour as expected. Although the embankment was made up of mixed soils and abrupt changes were noted in the soil



profile at the field site, the simulations from PLAXIS 3D were able to simulate the overall embankment behaviour and good agreement was observed between the field measurements and simulation results.

Authors' contributions

PB performed numerical simulation, conducted laboratory experiment and analysed the field data. DTB contributed in designing the embankment and provided the guidance throughout the execution of the project. SD performed laboratory experiment (including large scale direct shear test) and monitored the field data. All authors read and approved the final manuscript.

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Competing interests

The authors declare that they have no competing interests.

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